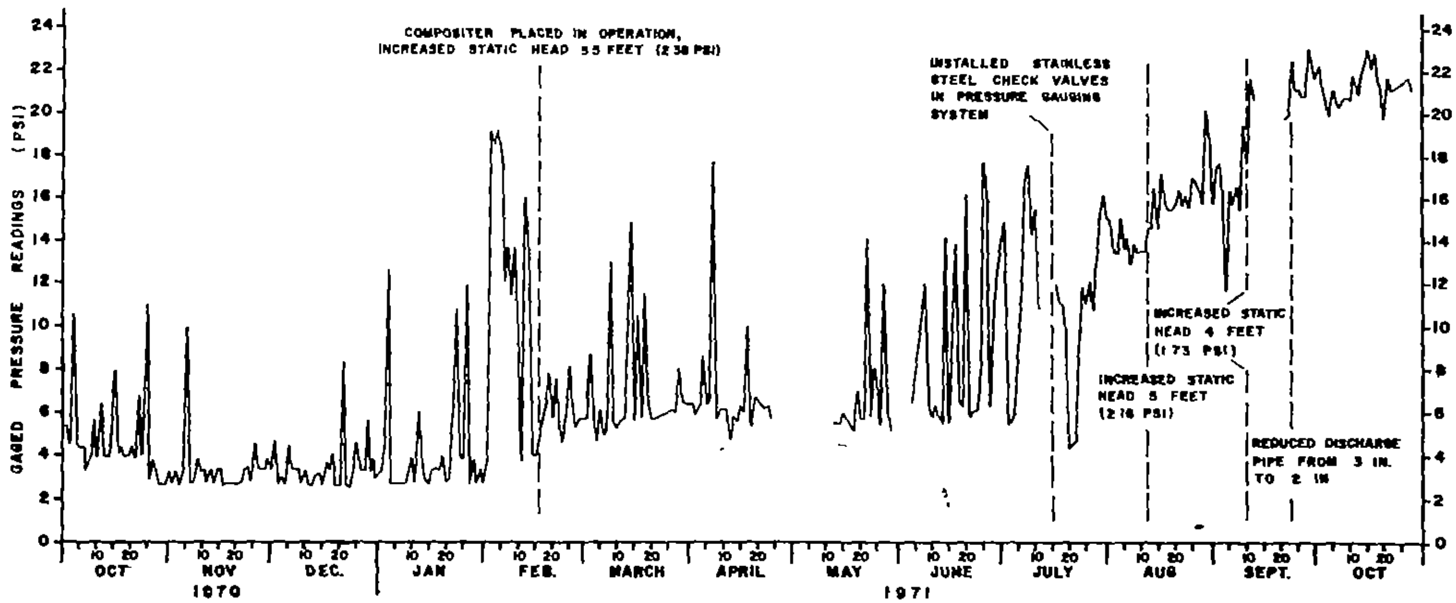
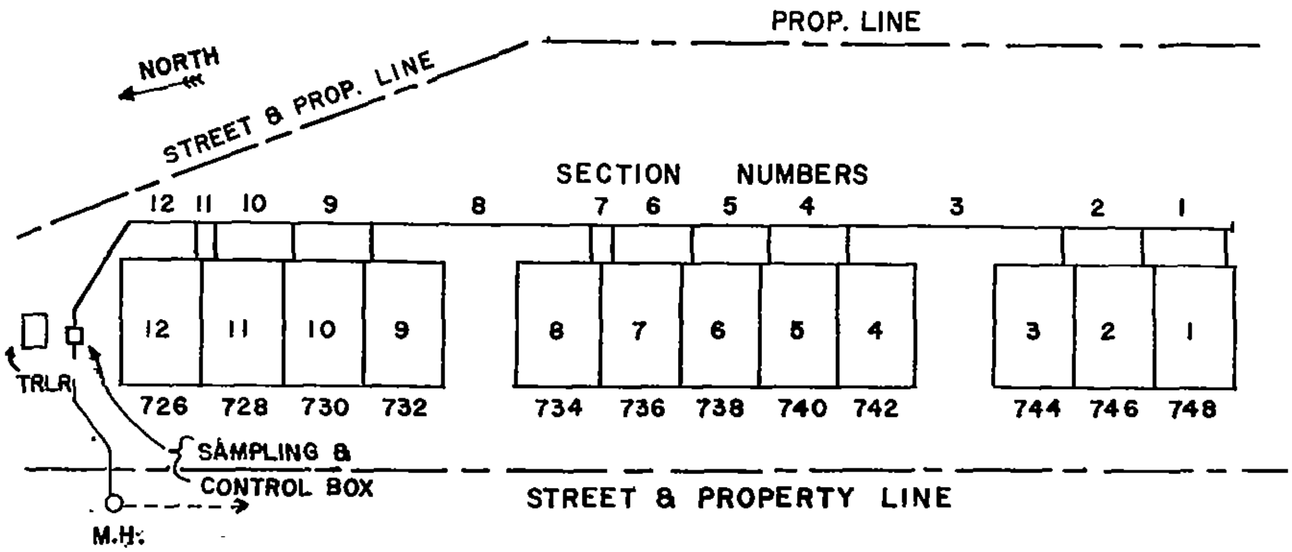


FIGURE 8

GAGED PRESSURE READINGS FOR UNIT NO. 2

FIGURE 9

ASSUMED AND ACTUAL FLOWS FOR THE PRESSURE MAIN



SECTION NUMBER	PVC-DWV PIPE SIZE	LENGTH OF SECTION (FT)	ASSUMED MAX. FLOW (GPM)	MAX. FLOW RECORDED (GPM)	ASSUMED MIN. FLOW IN 24 hrs (GPM)	DAILY FLOWS (GPM)		
						MAXIMUM	AVERAGE	MINIMUM
1	1.25"	19.7	15	15	15	15	15	15
2	2.0"	20.0	30	30		30	30	
3		59.2	45	45				
4		19.2			30	45		
5		19.5						
6	3.0"	19.6	60					
7		1.9		60	45			
8		58.5						
9		17.4	75					
10		19.5			60			
11		2.7						
12		81.0	90					

TABLE 3

SUMMARY OF COMPOSITE SAMPLE ANALYTICAL RESULTS

Parameter	Number of Samples	Mean*	Standard Deviation	Minimum Value	Maximum Value
5 Day Biochemical Oxygen Demand	57	330	53	216	504
Chemical Oxygen Damand	56	855	158	570	1450
Soluble Total Organic Carbon	6	140	49	21	225
Total Solids	55	681	87	526	928
Total Volatile Solids	56	476	84	336	706
Total Fixed Solids	56	205	63	57	355
Total Suspended Solids	56	310	77	138	468
Volatile Suspended Solids	56	274	84	78	440
Fixed Suspended Solids	56	36	48	0	268
Total Dissolved Solids	55	372	90	195	637
Volatile Dissolved Solids	55	201	62	22	372
Fixed Dissolved Solids	55	171	58	27	353
Organic Nitrogen**	53	29	12	7	76
Ammonia Nitrogen**	54	51	9	34	68
Nitrate Nitrogen**	38	0.1	-	--	--
Total Phosphate***	63	15.9	6.3	7.2	49.3
Particulate Phosphate***	50	2.8	0.9	0.4	4.2
Filterable Phosphate***	51	13.1	6.5	5.2	47.9
Total Ortho Phosphate***	32	8.7	3.9	1.3	17.9
Methylene Blue-Active Substances****	39	12.4	4.5	4	24
Grease	9	81	12.3	31	140
Settleable Matter ½ Hr.	56	14.5	6.1	4	37
Settleable Matter 1 hr.	56	15.0	6.2	4.5	38
Chlorides	38	52	4	41	61
Hardness	55	65	7.4	46	90
Alkalinity	9	198	8.1	185	209
pH	54	7.8	.3	7.1	8.7

* All values expressed as mg/l except pH
** As nitrogen
*** As phosphorus
**** As linear alkylate sulfonate

There are no existing standards for velocities dealing with the grease accumulation problem, even though velocities in the range of 2 fps to 8 fps have been used by some in designing wastewater pressure conduits. However, for a pressurized sewer system utilizing GP Units, a velocity range of 2 fps to 5 fps is hydraulically and economically preferable.

Extensive chemical analysis were performed (Table 3). The concentration of various pollutants in a pressure sewer system was found to be approximately 100% greater than those found in conventional systems. On a gm/capita/day basis the pressure sewer waste contained approximately 50% less contaminants than reported for conventional domestic sewage. Settleability tests show no significant differences when compared with conventional wastewater.

Therefore, the difference in the strength must be taken into account in designing treatment facilities for a pressure system.

Conclusions

The pressure sewer system, which included the usage of PVC Schedule 40 pipes and PVC-DWV fittings, functioned well for the duration of the demonstration project. Careful considerations must be given to the material used in backfilling pressure main trenches. A good engineering practice is to encase the plastic pipe in sand.

As for the GP Units, the functional specifications have proven to be appropriate. Even though the Prototype Unit exhibited low mechanical reliability, the Modified GP Unit operated to its expectations. Design modifications virtually eliminated all major malfunctions; that is, the 1" opening of the pressure sensing tube was increased to 3" and the pump was relocated so as to be positively primed.

The service record coupled with the "down-time" performance of the Modified Units was impressive, a 0.27% "down-time" value versus a 2.69% "down-time" value for the Prototype GP Units.

Both the pump size and tank volume were more than adequate to handle peak wastewater flows, so that no further design modifications are necessary in this area.

Therefore, in order to summarize the operational performance of the GP Units, a brief review of previously presented facts has been tabulated;

(1) Total Number of GP Operations for the duration of the project - 73,740 operations

(2) Average Operations per capita per day - 2.6

(3) Average Length of operating cycle - 57-74 sec.

(4) Electrical power consumption cost - 34¢/capita/year

In addition, based on the water consumption data, an average wastewater flow of 37 gallon/capita/day was computed. A comparison of the chemical analysis for the pressure sewer project versus the results obtained by others from the conventional gravity systems (Table 4) indicates a much stronger sewage, yet one that contributes 50% less pollutants on the per capita basis. Also, settleability tests indicated no significant difference between the pressure and conventional sewage.

Recommendations

It is recommended that pressure sewer systems be considered as available engineering technology for use where applicable. This recommendation is based on the high mechanical reliability demonstrated by the Modified GP Unit during this demonstration period.

TABLE 4

COMPARISON OF PRESSURE SEWER SYSTEM WASTE WITH CONVENTIONAL WASTE
(From "A Pressure Sewer System Demonstration" by Carcich et al)

Parameter	Conventional Gravity System Concentration mg/1	Conventional Gravity System Loading gm/cap/day	Individual Home Systems Concentration mg/1	Individual Home Systems Loading gm/cap/day	Pressure Sewer System Concentration mg/1	Pressure Sewer System Loading gm/cap/day
5 Day Biochemical Oxygen Demand	180	68	284-542	44-158	330	39
Chemical Oxygen Demand	400	150	540-882	82-205	855	102
Total Solids	700	265	788-1249	113-216	681	81
Total Volatile Solids	350	132	414-659	60-138	476	56
Total Suspended Solids	200	76	293-473	44-106	310	37
Total Dissolved Solids	500	189	-	-	372	44
Settleable Matter(1)	70	-	-	-	15	--
Organic Nitrogen	20	7.5	-	-	29	3.5
Ammonia Nitrogen	11	4.2	48-92	8-16	51	5.9
Total Nitrogen	31	12	61-121	11-20	80	9.4
Total Phosphorus	11	4	15-21	1.9-5.7	16	1.93
Chloride	23	8	-	-	52	6.1
Crease	40	15	33-95	6.1-28	81	10.35
Flow	-	100 gal/cap/day	-	24-78 gal/cap/day		32 gal/cap/day

(1) Expressed as mg/1

The hydraulic design is a very critical facet of any new pressure sewer system. Some grease accumulation can be expected within the pressure pipes. However, critical hydraulic design will limit any excessive grease accumulation and at the same time, will offer the most economical system to the customer. Installation of down-stream clean-outs should be considered as part of good engineering design practice.

Commercially available shut-off valves should be installed in order to isolate the GP Unit from the pressure system during maintenance or repair work. Installation of air relief valves and curb stops should be common practice in as much as the pressurized sewer system is, in many respects, similar to a water distribution system.

Extensive power failures are not very common occurrences in this country. However, since some areas are often hit by power failures due to weather conditions and other uncontrollable factors, it should be noted that GP Unit's tank affords up to 8 hours of wastewater storage, which would be sufficient to accommodate short duration power failures. If historical data from the local power utility dictates a need for an overflow system due to frequent and extensive power failures, then inexpensive cesspools or existing septic tanks can be utilized to receive the wastewater overflows from the GP Units.

At the outset of this demonstration project, emphasis was placed on the fact that the pressure sewer system concept was not meant to replace the conventional wastewater collection system. Rather, the pressurized system is to be used as a supplemental engineering tool for optimizing any wastewater collection system. The economic analysis and sound engineering practice will dictate the extent of the usage of the pressure sewer system instead of the conventional gravity system.

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SECTION V

APPLICATION OF MICROSTRAINING TO COMBINED SEWER OVERFLOW

by

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Combined sewer overflow is a mixture of stormwater and sanitary flow. The special problems of dealing with this flow are due almost exclusively to the stormwater component. Thus, these remarks should apply equally well to overflow of separate storm sewers.

The two components - stormwater and sanitary waste - are somewhat similar in composition. Both contain suspended solids, BOD, and coliform concentrations equal to many times the usual secondary effluent standards. On an annual basis our eleven acre drainage area produces some 9,000,000 gallons of sanitary flow and about 3,000,000 gallons of storm runoff.

The flow rate of stormwater runoff, however, is very high and widely variable. At our site, we have monitored several storms a year where the runoff rate is over 400 times the mean dry weather sanitary flow. It is the flow rate aspect of combined (and separate) sewer overflow that requires a totally different approach when treatment is considered.

Only recently have we become aware of the magnitude of the possible pollutional load from stormwater runoff and have considered treating it. It is not surprising that there is a considerable difference in opinion as to what a stormwater treatment facility should be able to do. The two basic dimensions of a combined sewer (or separate storm sewer) overflow treatment facility are:

(a) The instantaneous flow rate it can handle, and

(b) the amount of each type of pollutant it can remove.

In our studies we have used a flow rate of 2.0 cfs/acre (1.34 mgd/acre) as the required instantaneous capacity of the treatment facility. This runoff rate would require (at a runoff coefficient of 0.4) 4.5 inches per hour rain intensity. At our site we have this intensity sustained for about 15 minutes every 10 years. Analyses of very large drainage areas such as the Boston and Chicago stormwater tunnels where rainfall does not occur over the entire area simultaneously, and where there is tremendous surge volume within the sewer (tunnel), have led to the adoption of a flow rate of 0.2 cfs/acre (0.13 mgd/acre) based on the area of the entire basin. Less understandable is the adoption of low (0.2 - 0.3 cfs/acre) instantaneous design rate for the treatment of combined sewer overflow from small drainage areas of 100 acres or so. Additional experience will permit the selection of realistic design rates for each situation.

It has been suggested that flow equalization basins be included above ground as part of the overflow treatment facility to reduce the peak instantaneous flow rate. Above ground, flow rate equalization basins by themselves may be an attractive scheme of treating overflows, providing space at low cost is available. In this scheme, the peak overflow rate is reduced to a rate where the existing interceptor sewer and sewage plant can handle it as an alternative to an on-site combined sewer treatment facility. Although the annual stormwater volume is some 35% of the sanitary volume, only some 15% additional flow rate capacity would be required.

Flow equalization is most attractive where the subsequent treatment techniques are very expensive on a dollar/cfs peak capacity basis. Flow equalization is essential where the subsequent treatment techniques cannot accept sudden starts and stops or rapid changes in flow rate of several hundred times the dry weather flow variation.

The extent of treatment to be required on combined sewer overflow is at present not standardized. It is not certain what form regulations will take. As will be seen later the familiar "percentage removal" type regulation would be most inappropriate for this problem. Much more work and study must be completed before it can be decided whether it is necessary or consistent with the cost to design overflow treatment facilities for a 25 year return storm or a 5 year return storm.

With current practice, the combined sewer overflow regulator is adjusted to overflow when the rate exceeds perhaps 3-5 times the mean dry weather flow. Thus, the composition of the combined sewer overflow is 1 part sewage to at least 1-1/2 parts of storm runoff. Frequently the composition is over 100 parts of storm runoff to 1 part of sanitary flow. In any event, when significant overflows occur, the composition of the overflow water is determined almost exclusively by the composition of the storm runoff.

The wide range of contaminant levels in the combined sewer overflow reflect the breadth of the range in the storm runoff.

The contaminant level in the combined sewer overflow observed in our site is shown in Table 1.

Table 1

Contaminant		Minimum	Mean	Maximum
Suspended solids	mg/l	15	100	700
BOD_5	mg/l	8	800	3,000
Total coliform cells/100 ml		1,000	1,000,000	3,000,000

Previously we had found (during the fall and winter storms) that, in general, the contaminant concentrations were higher on the bigger storms particularly in the case of the suspended solids. Recently, however, (during spring and summer storms) we found little relation between storm intensity and contaminant levels. The BOD and coliform content of overflow do not seem to have any relation to storm intensity but do seem to have an annual variation. Each drainage area has no doubt a unique combination of features which will influence the character of the stormwater overflows. Our experience, however, has been paralleled by the reported observations of others. They find that sustained higher contaminant concentration levels are as likely if not more likely to occur in large overflows from the bigger storms as from the smaller overflows from less intense storms.

Thus, the treatment design criteria and the regulations must, for the present, assume that maximum overflow contamination concentration will exist at design peak flow rate. More work is needed on this aspect.

To attack a given combined sewer overflow situation, the first step is to predict the peak rate-duration and frequency of the actual overflows. With these predictions at hand a decision to treat all storms of less than a certain return frequency must be made more or less arbitrarily. One method of arranging the storm flow data is that used by Dow (2). See Figure 2 from that report. Note that treating about one-third of the peak flow observed over an 8 year study would treat some 98% the total annual flow.

The benefit of flow equalization can be evaluated for the storms to be treated. That is, the relation between equalization basin volume and the reduced peak rate can be ascertained. This work might be extended to, say, 60 minutes, which will be the residence time of some of the actual treatment techniques. We will return to this flow rate consideration after we look at the degree of treatment needed.

There is paucity of information regarding the impact of combined sewer overflow contaminants on the receiving stream. It seems that the pounds of suspended solids discharged per year would be an important criterion.

It is not known how much greater impact these solids would have when they are discharged in slugs of approximately 40-60 hours annual duration. If it is found that the instantaneous rate of solids discharge is significant, the regulations may be phrased in terms of maximum pounds per hour. This is a very complex problem and the methods of considering it have not been developed.

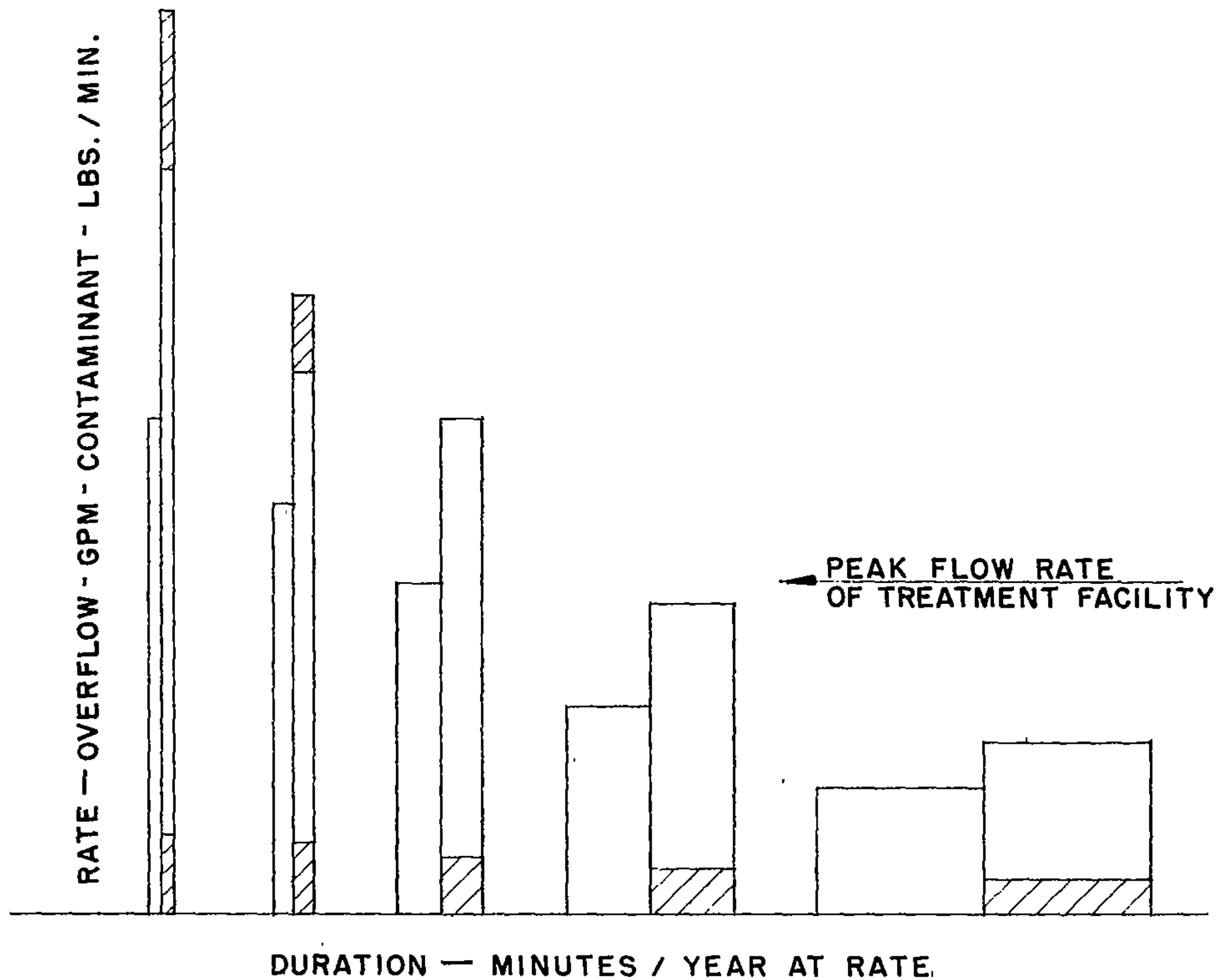


FIGURE I
DISCHARGE RATE - ANNUAL VOLUME
OF OVERFLOW - CONTAMINANTS

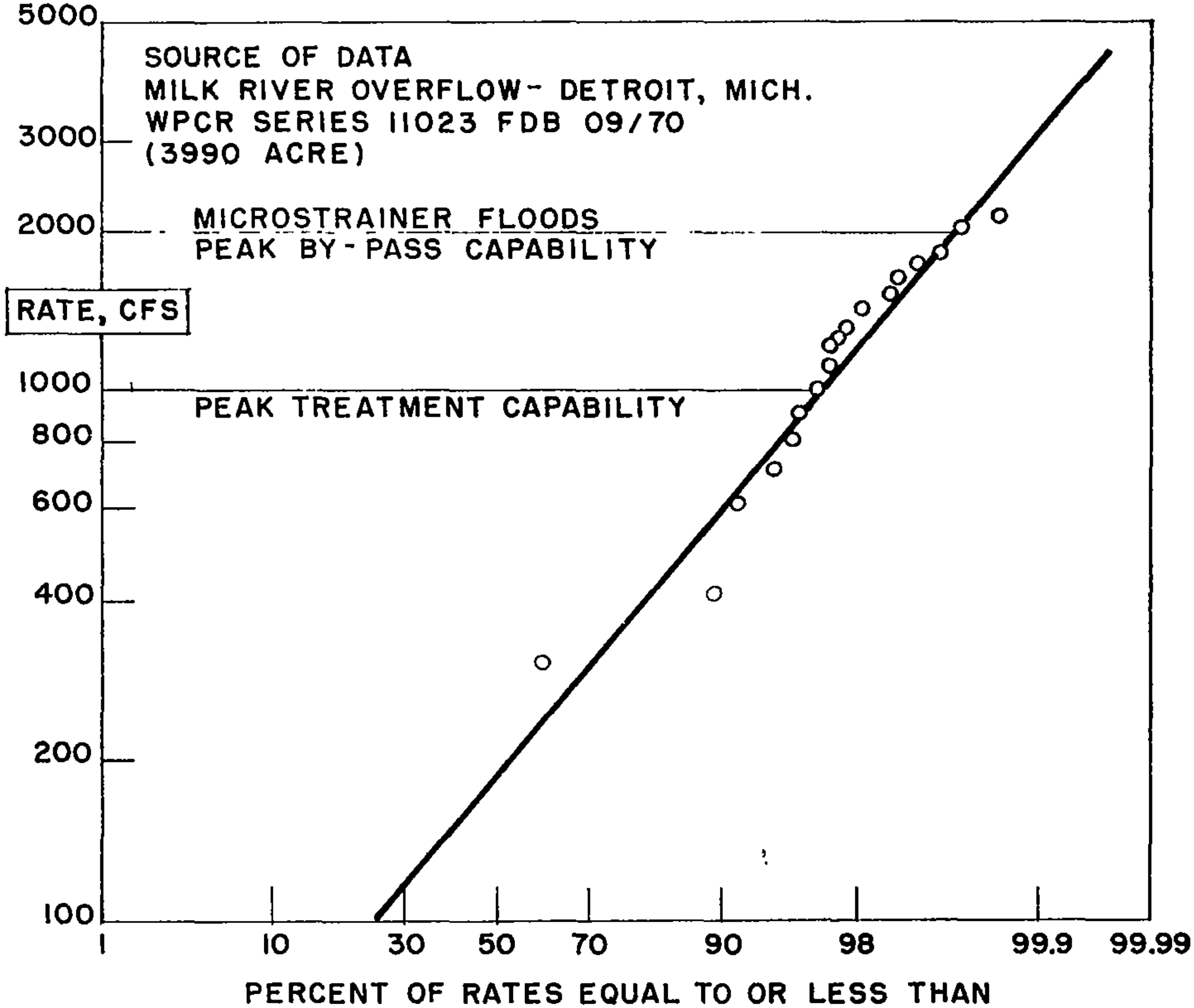


FIGURE 2
DISTRIBUTION OF STORM OVERFLOW RATES (1960-1968)

The potential pollutant load of untreated combined sewer overflow during a big storm is: (Overflow Rate) x (Pollutant Concentration; e.g., S.S.).

The potential load can be reduced by treatment to a lower level, depending upon the design of the treatment facility as follows:

(Overflow Rate-Peak Capacity) x (Pollutant Concentration) plus

(Peak Capacity) x (Pollutant Leakage).

Figure 1 is a preliminary attempt to illustrate this relationship in a stylized manner. The bars represent overflows in increments of magnitude. The height of the bar represents the magnitude of the flow (the left of the pair) and of the instantaneous contaminant flow; e.g., pounds of suspended solids per second. The width of the bar represents the duration of flow of the indicated magnitude in minutes per year. The area of the bars then represent overflow volume per year at indicated rate (left of pair) and the pounds of contaminant per year. The shaded area at the bottom of the solids bar represents the solids leaking through treatment facility and entering the stream. An arbitrarily selected design peak flow rate for a treatment facility is shown. The shaded area on the solids bars representing the biggest storms shows the additional solids entering the stream by direct bypass of the facility.

The amount of the annual contaminant load to the river of the design parameters - peak flow capability of the facility and the leakage through the facility can be seen. Also, the instantaneous rate of contaminant discharge can be seen.

Figure 2 shows another way to consider the overflow rate-annual duration data.

In the previous application section, I have attempted to show the importance of Peak Flow Rate Capability of a combined sewer overflow treatment technique (s). Also I tried to show the importance of Contaminant Level Removal Capability of treatment techniques at design (peak) rate and below design rates.

The announced subject of this paper is a description of the capability of the Microstraining technique in this service.

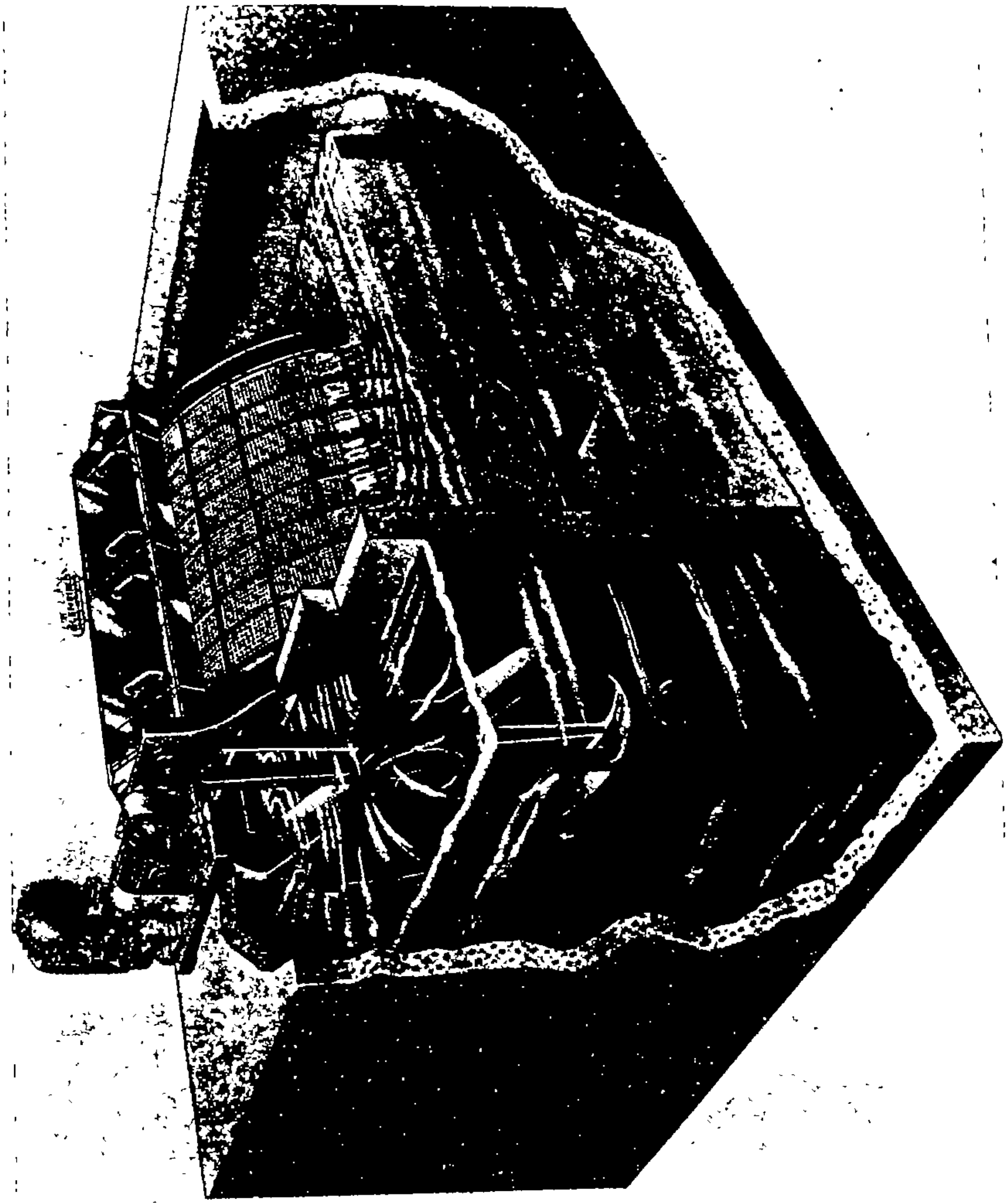
Figure 3 is an isometric drawing of a microstrainer. A microstrainer is a rotating drum fitted with fine screen. For stormwater the screen used is what we call Mark 0, a stainless steel Dutch twill screen with 600 x 125 wires per inch yielding about 23 micron (1-1/2 millions of an inch) apertures.

The stormwater enters the open end of the drum and passes through the screen into the outlet chamber and then to waste. The suspended solids are retained by the screen. As the drum rotates, the screen with a mat of retained solids on the inside is brought up and under a row of backwash jets which wash the solids off into a hopper and thence to disposal. The backwash water requirement is about 1-1/2 gpm per foot of drum length which is a fraction of a per cent of the thruput capability. The solids-rich backwash water stream is small - less than the DWMF - and can easily be sent via the interceptor to the sewage plant, for smaller CSO facilities, or disposed of locally. The backwash water source can be repumped microstrained CSO or preferably city water on small unattended satellite facilities.

The flow of water through the screen is motivated by the difference in level inside the drum over the level outside the drum. In conventional applications of Microstraining this differential is about 6 inches. At this differential

Figure 3

Isometric Drawing of a Microstrainer



the Mark 0 screen will pass only about 6-8 gpm/ft^2 of gross submerged screen area. It might be noted here that the flow capability is not based upon the gross area of the drum but rather upon the open submerged area. That is, that area of screen unimpeded by hold-down straps which lie below the liquid level inside the drum. There is considerable difference in the per cent submergence attained and the per cent unimpeded area in currently available microstrainers and the percentages vary a little from size to size. In the Current Crane design for a 10' dia x 10' long drum, the per cent submergence is 83% and the per cent unimpeded area is 94%. The Glenfield-Crane (older) design we are using has only 83% unimpeded area and was adapted to achieve 83% submergence. Some competitive designs have lower percentage submergence and unimpeded area.

For stormwater service we use much higher differentials, up to 24", and have achieved flow rates of up to 45 gpm/ft^2 of gross submerged area (i.e., 54 gpm/ft^2 of unimpeded submerged area) with very high removals.

The following remarks will be based upon 35 gpm/ft^2 of gross submerged area (42 gpm/ft^2 of unimpeded submerged area). Also, these remarks will be based primarily on the use of a microstrainer as a satellite station for treatment of CSO; i.e., located at the point of overflow so that no additional sewerage is required.

Perhaps the best way to describe a microstrainer CSO facility is by an example.

A present-day Crane 10 x 10 has 314 gross sq ft of screen area of which 245 sq ft is unimpeded and submergible. Such a machine can treat some

10,500 gpm or 23 cu ft/sec of any of the combined sewer overflows we have seen in 16 months of study. Our example will be a facility with two such machines in parallel. (As previously mentioned, the 46 cfs (30 mgd) flow capability of these two machines would be required by a drainage area of from 24 to 240 acres depending on many factors unrelated to the microstrainer.)

Any CSO treatment facility will require a coarse bar screen. The space for and the cost of a travelling bar screen have been included in this example facility. Almost certainly any CSO treatment facility will be sized to treat something less than the peak storm that will occur in the life of the equipment. Thus, a bypass arrangement is required to divert the flow in excess of the peak capacity of the treatment equipment without interfering with the capability of the equipment to treat its peak flow. This consideration may be less important with Microstraining than with other techniques. A microstrainer will flood; i.e., untreated water will overflow the washwater hopper at inlet levels 3" or so above the design level at peak design flow rate. The microstrainer cannot, however, dump previously removed solids into the effluent under excess flow conditions. The space for and the cost of a bypass weir and channel suitable to divert excess flow equal to the design flow have been included in this facility. That is, this facility can accept 92 cfs, treat 46 cfs without hinderance, and bypass the reaminder to the receiving stream, or rather to the disinfection chamber, and then to the stream.

The bar screen-microstrainer facility with flumes and chambers for bar screening of 92 cfs, Microstraining of 46 cfs, and bypass of 46 cfs will occupy a ground area of 30 x 40 ft x 10 ft deep. The facility area of 1200 sq ft of ground area is 1/35 acre or about 1/1000 to 1/10,000 of the drainage basin. The liquid volume of the facility is about 9,200 ft^3, or 200 sec residence, at peak flow. The head loss through the facility is about 3 ft during peak flow. While 3 ft is the minimum head required during a storm, ideally there should be 10 ft of head available so that the facility can be drained by gravity after the storm. Otherwise, a small (3 hp) sump pump will be required.

The chamber will be comprised of about 2,500 sq ft of concrete walls and 1,200 sq ft of floor, and to put it below ground will require about 600 yards of excavation.

The microstrainer section should be housed and kept above freezing. The recommended building then would be about 16' x 40' x 18' high. The individual microstrainer units weigh about 13,000 pounds and an I beam craneway should be provided for installation and maintenance. An insulated Butler Building of this size is included in the cost data.

To keep the microscreen in condition to operate when needed it must not be allowed to become dry while soiled. The recommended procedure for combined sewer overflow service then is following a storm to drain the chamber, continue the backwash of the slowly rotating drum using city water as washwater for several hours and then stop the drum and the backwash water.

Also, for sustained dry periods the drum can be rotated slowly for short periods at intervals under backwash jets and the UV lights. The program controls for carrying out this maintenance operation automatically are included in the cost data.

The cost of a complete facility installed, less land and engineering, was estimated to be \$195,000 in 1969 dollars. This investment represents an annual capital charge of about \$19,500/year to be applied to the facility. This annual capital charge is, by far, the major cost for Microstraining (or other techniques) for combined sewer overflow. This cost applied to the drainage area represents about \$80 to \$800 per acre at peak design rating of 0.2 and 2.0 cfs/acre respectively.

The effect of scale on the cost of a facility can be seen in Figure 4.

The utilities required for the two machine facility include about 50 gpm of city water. The electrical power demand is for two 5 hp drum drive motors, a 3 hp sump pump, if required, a 5 hp drive for the automatic bar screen rake, and for lighting and controls - about 25 kilowatt connected load in all. With 50 overflow events a year (we see only 40), and several hundred short, dry weather periods of operation, the running time then will be 280 hours a year so that the annual power consumption will be 7,000 kwh/year or abour \$140/year. Similarily, the city water consumption will be about 14,000 gallons/year most of which is consumed during rainy weather.

The microstrainer is automated. At onset of storm overflow the liquid level in the inlet channel rises and actuates a level switch which starts the microstrainer drum motor, the backwash jets, turns on the UV lights, and the bar screen rake drive.

The microstrainer drum speed controls regulate the speed of the drum in accordance with the difference in liquid level across the screen which is roughly proportional to the flow rate. All of the combined sewer overflow passes through

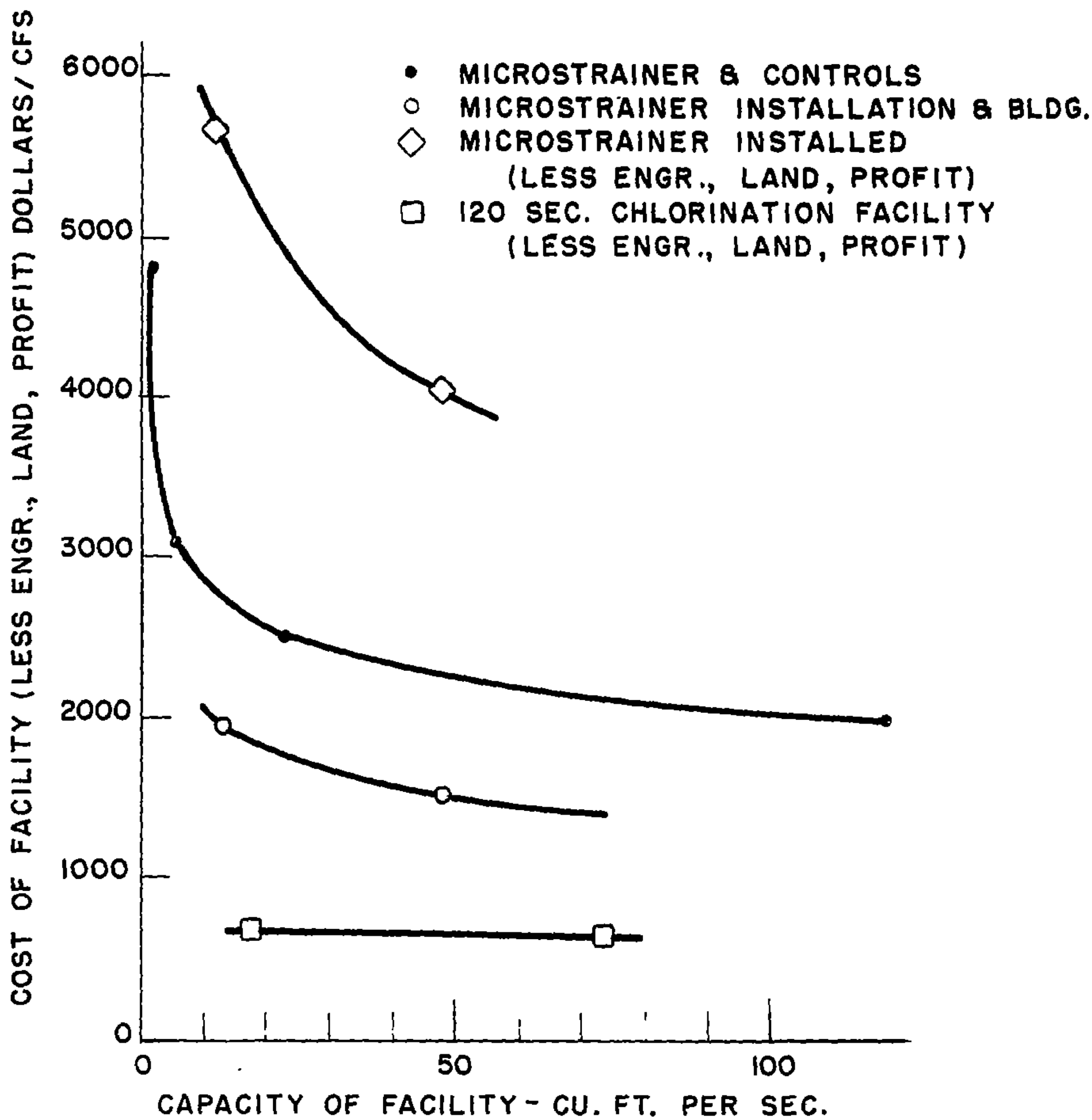

MICROSTRAINER & CONTROLS
MICROSTRAINER INSTALLATION & BLDG.
MICROSTRAINER INSTALLED
(LESS ENGR., LAND, PROFIT)
120 SEC. CHLORINATION FACILITY
(LESS ENGR., LAND, PROFIT)
COST OF FACILITY (LESS ENGR., LAND, PROFIT) DOLLARS/ CFS
6000
5000
4000
3000
2000
1000
0
0
50
100
CAPACITY OF FACILITY - CU. FT. PER SEC.

FIGURE 4

the drum. If the storm flow should exceed the peak design rate of the machines (i.e., cause a differential in excess of 24") the excess water overflows the bypass weirs and flows directly to the receiving stream or to the disinfection facility and then to the stream. At the end of the storm, the program controls continue the operation of the microstrainer, sump pumps, etc., until the chamber is drained and the screen is clean and then shut them down. The instant readiness and the very low residence volume of the Microstraining technique permits unattended operation with very simple controls. Our equipment ran on all storms under automatic controls. It was unattended during the first part of all storms. No trouble was observed.

The labor required for a facility would be weekly inspection and routine maintenance visits (i.e., lubrication, etc.) and it is believed that a two man crew could accomplish this in 2 hours. The labor cost would be the cost of 104 hours, or at $2.50/hour, $260/year.

Maintenance supplies, replacement parts, and maintenance labor (in addition to operation-routine maintenance labor) should not exceed 1% of the facility cost per year. We have no long-term experience on the screen life at high differentials, however, it is believed that the original screen will serve for 10 or more years in stormwater service. The cost of rescreening a 10 x 10 is about $5,000. Our experience over a 3 year period has indicated a maintenance cost of less than 1% of facility cost, even if a screen change every 10 years is anticipated.

In summary, the annual cost of a facility having 490 sq ft of open submersible area (capable of treating 45 cfs) would be:

Capital charge @ 10% of installed facility cost (less land and engineering)	\$19,500
Utilities - electric power and city water	200
Routine labor	250
Maintenance and supplies @ 1% of installed facility cost	1,950
	\$21,900

The annual cost of installing and operating a dual 10 x 10 microstrainer facility is \$22,000/year. Such a facility will accept 92 cfs and treat 46 cfs. Depending on conditions previously discussed, such a facility would serve a drainage area of from 24 to 240 acres.

The suspended solids removal performance of a microstrainer on stormwater follows a pattern that will seem strange to engineers accustomed to other liquid-solid separation techniques such as settling or granular bed filtration.

A large portion of the first increment of solids applied to the screen leak through before the mat is established. Most of subsequently applied solids are retained as shown in Figure 5. Thus, those conditions that contribute to high solids loading; i.e., high potential pollution make for high removals. These conditions are high flow rate, high stormwater solids concentration and low drum speed. It may be repeated that the higher the flow rate and the higher the influent solids, the lower the effluent solids. This latter relation is shown in Figure 6 and Figure 7.

The suspended solids in the stormwater at our site exhibited a surprising

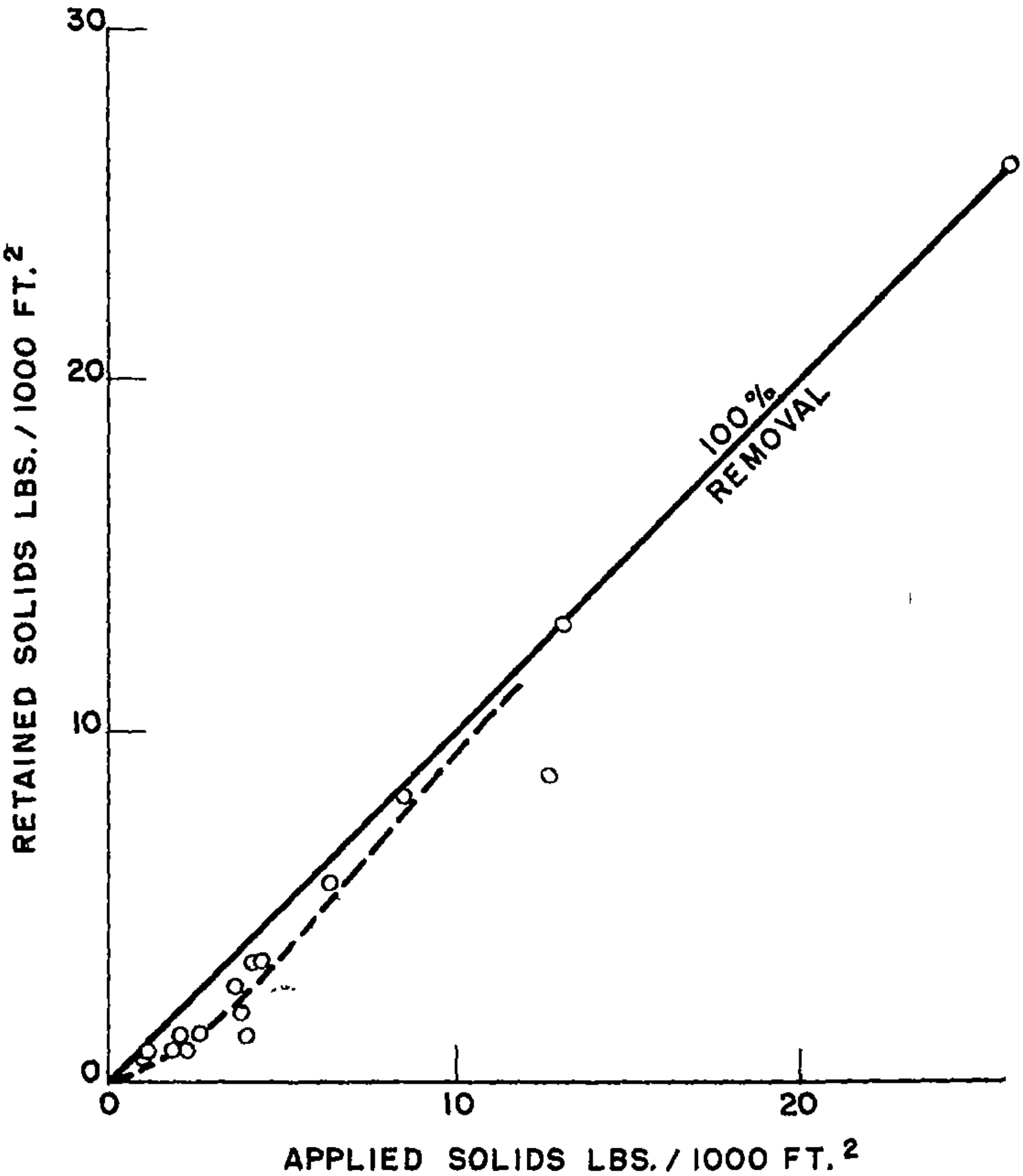


FIGURE 5
STORM WATER SOLIDS APPLIED VS. STORM WATER SOLIDS RETAINED

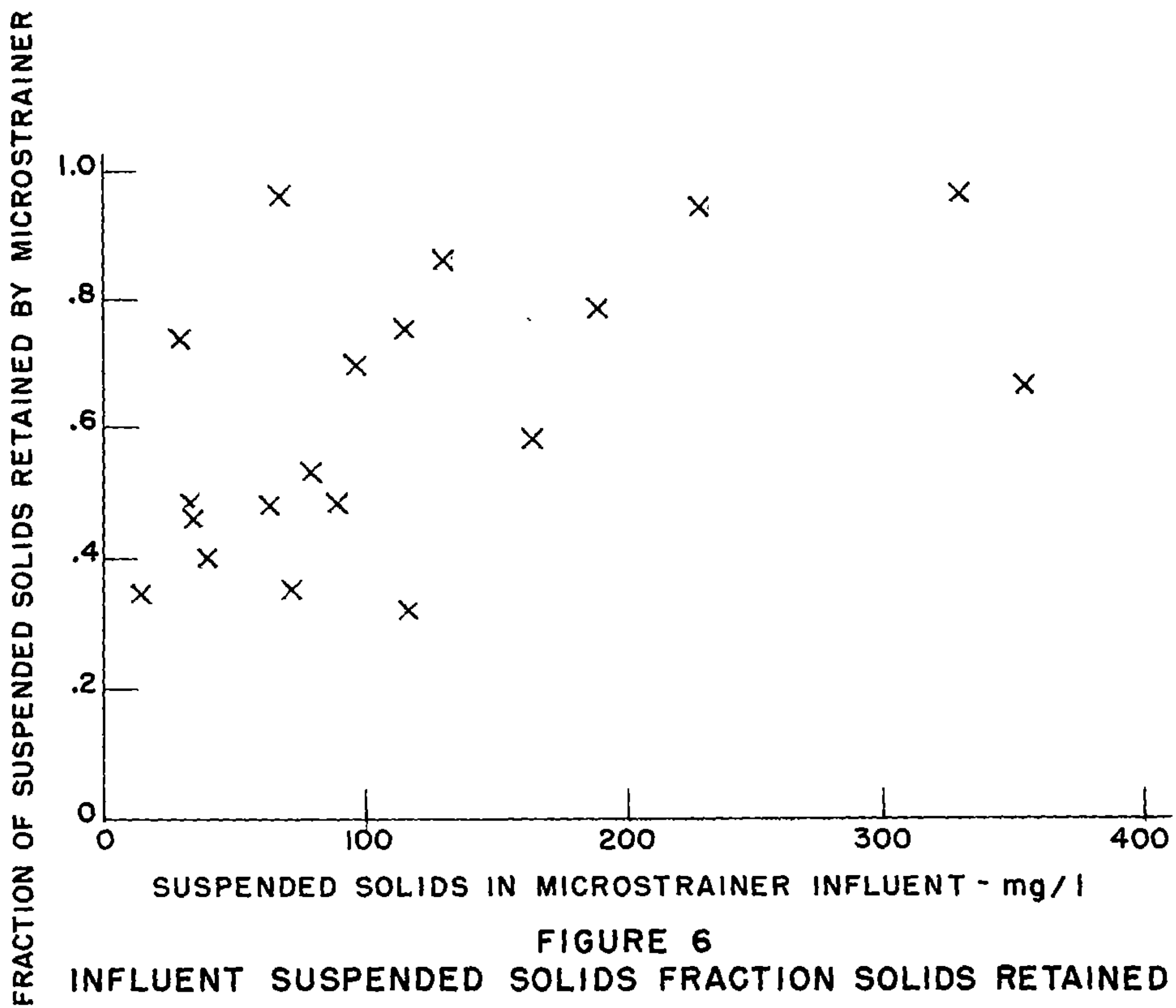


FIGURE 6
INFLUENT SUSPENDED SOLIDS FRACTION SOLIDS RETAINED

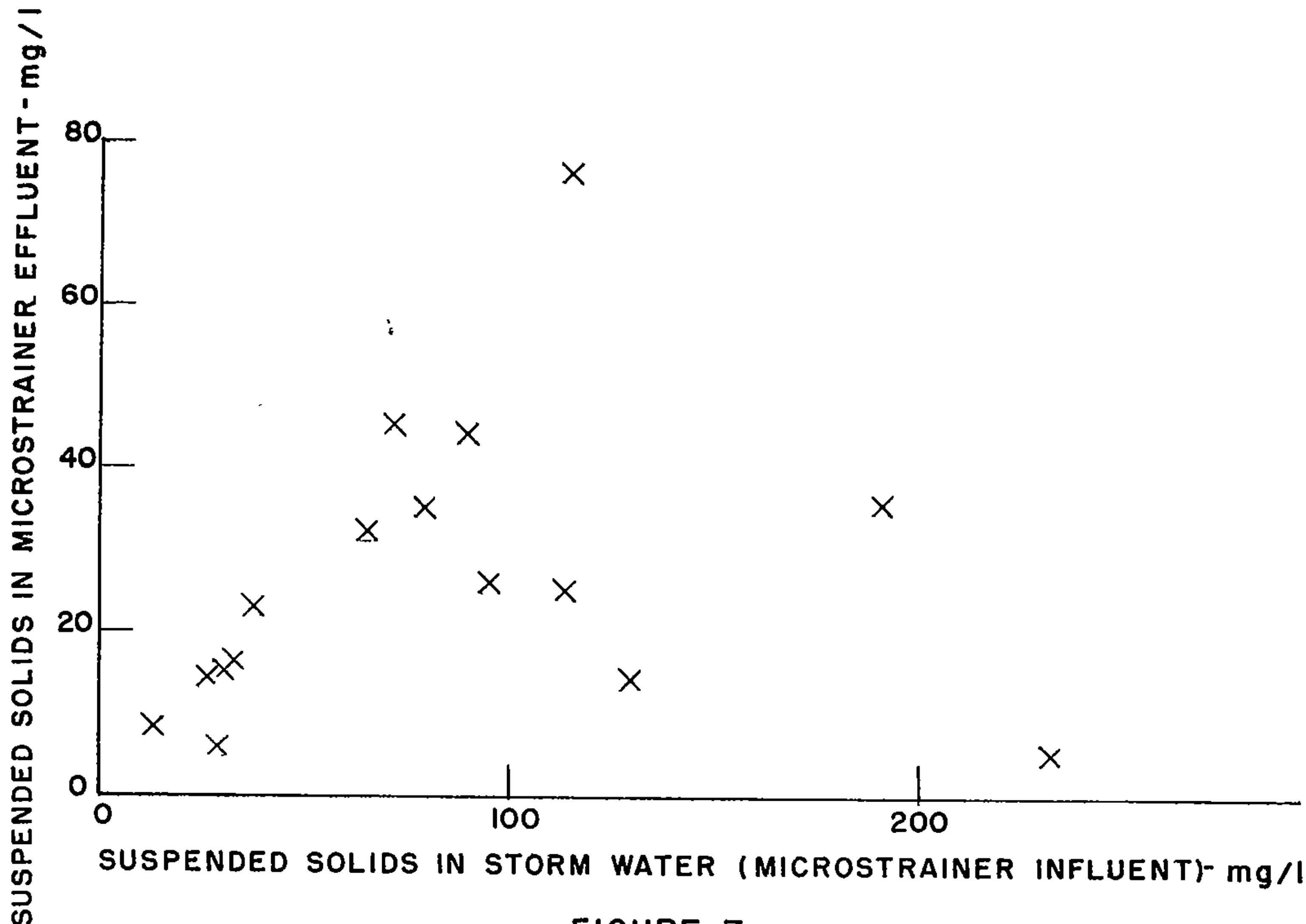


FIGURE 7
INFLUENT SUSPENDED SOLIDS VS. EFFLUENT SUSPENDED SOLIDS

characteristic. The greater the concentration of solids the easier they were strained out. The permeability parameter is the flow rate possible at unit head loss; i.e., one inch of water head loss per inch of mat thickness. The units of this parameter, borrowed from oil well practice, are inconvenient for Microstraining since buildup consists of mats of a few thousandths of an inch. In any event, this permeability is a measure of the flow capacity of the machine within the differential limitation imposed by the screen strength.

In summary, we have found in two studies totaling about 22 months of operation at one site that the microstrainer will reduce suspended solids from 50-700 mg/l down to 40-50 mg/l at flow rates of 35 to 45 gpm/ft^2 of gross submerged screen area; i.e., 42-54 gpm of unimpeded submerged area. These flow rates have been routinely achieved within an arbitrary limiation of 24" of water differential between inlet and outlet liquid levels.

The removal of organic and other oxygen demanding material is shown on Table 2 to be 25-40%. This removal is confirmed by BOD_5, COD and TOC measurements performed by the Standards Methods with and without a maceration pretreatment in a Waring Blender. The advantage of this pretreatment is covered in the formal report on this work.

The Microstraining had little or no effect on the coliform content of the stormwater.

Table 2

Effect of Microstraining on Organic Matter as Indicated by Several Test Methods

Change in Content of Organic Matter by Microstraining	BOD_5		COD		TOC	SVS	DVS
	raw	macerated	raw	macerated	macerated	raw	raw
<u>No Changes</u>[x]							
Number of Instances	3	3	2	1	3	5	6
<u>Increase</u>							
Number of Instances	1	1	2	+2	1	0	1
Average of Increases mg/l	13	14	2500	4010	2300	0	33
Average of Increases %	100	100	350	260	250	0	100
<u>Decrease</u>							
Number of Instances	3	5	5	7	4	4	1
Average Decrease mg/l	129	310	516	380	126	37	33
Average of Decreases %	35	37	24	32	38	64	67

[x] All changes in concentration upon passage through microstrainer of 10 mg/l or 10% or less are considered no change.

The advantages of the Microstraining technique for suspended solids removal are:

1. Instant readiness and low residence volume permit simple automation for unattended facilities at remote locations.
2. Instant readiness and very high flow rate capability/unit equipment cost permits installation without flow equalization basins.
3. The low head loss - 3 ft - through the entire Microstraining facility will generally eliminate the need for repumping.
4. The removal performance of Microstraining, where highest removals, both absolute and percentage-wise, are achieved at highest flow rates and highest suspended solids loadings, is particularly suitable for the conditions existing in combined sewer overflow service.
5. The excess flow bypass is an integral part of a microstrainer facility and eliminates the need for this necessary feature as an appendage.
6. The very high flow rate capability and low residence volume permit Microstraining to be the lowest cost solids removal technique - less than $500/year per cfs capacity.

ACKNOWLEDGEMENT

This work was conducted with the City of Philadelphia in two phases, (1) under a contract from the Environmental Protection Agency to the Cochrane Division of the Crane Co., and (2) under an EPA grant to the City of Philadelphia. The efforts of City personnel were under the general direction of Carmen Guarino, Water Commissioner, with William Wankoff and M. Lazanoff, serving as Project Director and Laboratory Director. J. Radzuil headed the City's R and D Department who also lent valuable assistance.

The assistance and guidance of these people are gratefully acknowledged.

The overall guidance and helpful advice of Richard Field, Project Officer, EPA, Edison, New Jersey, were most valuable.

SECTION VI

HIGH-RATE MULTI-MEDIA FILTRATION

by

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GENERAL

The nature of combined sewer overflow, i.e., a highly pollutional, high volume discharge, requires a relatively high rate treatment process for economical pollution control. Deep bed, high rate filtration, a new development in the field of industrial wastewater treatment, has demonstrated favorable cost-efficiency factors when dealing with high volume wastewater discharges, especially where suspended solids comprise one of the principal contaminants. Thus, it was felt that such a process, which currently has significant applicability and usage in the steel industry, might prove an effective and efficient solution to the treatment of combined sewer overflows.

To evaluate the applicability and effectiveness of the high rate filtration process in removing contaminants from combined sewer overflows, a testing program was undertaken at Cleveland's Southerly Wastewater Treatment Plant, beginning in 1970. The work was undertaken by Hydrotechnic Corporation, Consulting Engineers, New York, New York, under the sponsorship of the Office of Research and Monitoring, USEPA.

The City of Cleveland ranks seventh in the nation in total area served by combined sewers (44,000 acres), and is fourth in population served by combined sewer systems (1,000,000 persons). As can be expected, Cleveland has a very serious problem of combined sewer overflows.

TESTING PROGRAM

The two major process units or equipment units in the proposed treatment system are the drum screen and the deep bed, high rate filter. The function of the screen is to remove coarser material (fibrous type, etc.) that would impede the filtration operation. Construction of a full scale treatment plant employing the process sequence under study would require design parameters for the screen and for the filtration process. The major criteria for the screen are screen type, screen mesh, and hydraulic loading.

The filtration system, which is the heart of the overall process sequence, can be characterized and described by the following parameters:

Media composition	Length of filter run
Media depth	Head loss
Filtration rate	Backwash water volume
Coagulant addition	Backwash procedure

A definition of these elements allows the construction of a full scale facility.

Testing equipment at Southerly included a drum screen, two 5,000 gallon storage tanks, lucite filter columns of four (4) and six (6) inch internal diameter, and chemical and polyelectrolyte feed equipment. (Figures 1 and 2)

The testing program evaluating the filtration components of the proposed system was conducted primarily in two phases: first, evaluation and selection of system media and filtration rates, and secondly, optimization of the filtration process via coagulants and polyelectrolyte addition prior to filtration.

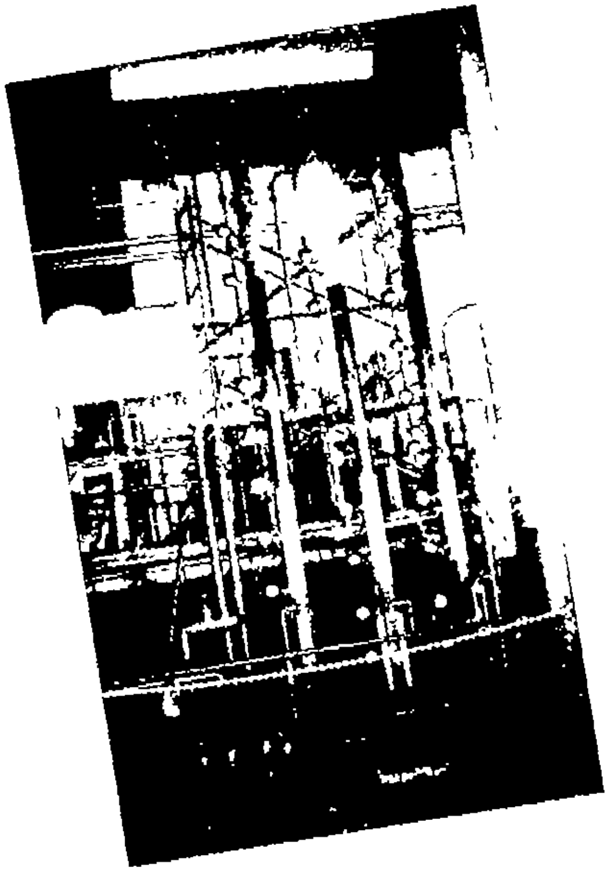

Figure 1
Lucite Filter Columns

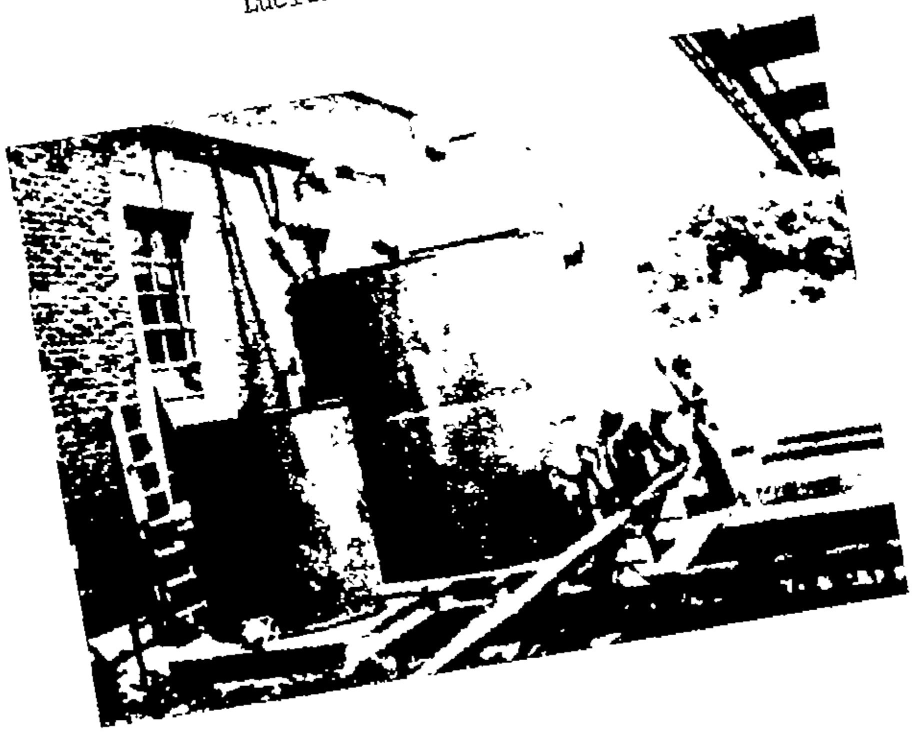

Figure 2
Drum Screen and Storage Tanks

filter run, and backwash procedure.

TEST RESULTS

The recommended system is a drum screen (No. 40 mesh screen element) followed by a deep bed, dual media filter (five feet of No. 3 anthracite over three feet of No. 612 sand). Sixty-nine pilot filtration runs were performed in 1970 and 1971 utilizing this system. Polyelectrolyte feed is an essential and critical part of the system to achieve optimum treatment efficiencies. Data utilizing coagulants ahead of filtration showed inconsistency in treatment efficiencies and at the present stage of development, polyelectrolyte feed alone appears optimum.

The proposed system, with addition of appropriate polyelectrolyte, achieved the following treatment performance:

Filtration Rate (gpm.sq ft)	Average Removals(%) Suspended Solids	BOD	Phosphorous
8	96	43	66
16	95	40	57
24	93	40	46

The average influent suspended solids concentration ranged from 50 to 500 mg/l and the average influent BOD concentration ranged from 30 to 300 mg/l. Effluent levels at 24 gpm/sq ft with polyelectrolyte addition were 15 mg/l suspended solids and 22 mg/l BOD, respectively. (Figures 3, 4, and 5)

HIGH RATE FILTRATION INSTALLATION

Combined sewer overflows would be conveyed from an automated overflow chamber, or chambers (in case the centralized filtration system is for many overflow points), to a low lift pump station.

Filtration media evaluated included: four or five feet of anthracite over three feet of sand. The characteristics of the media are indicated as follows:

Media	Effective Size	Uniformity Coefficient
No. 4 Anthracite	7.15 mm.	1.42
No. 3 Anthracite	4.0 mm.	1.5
No. 2 Anthracite	1.78 mm.	1.63
No. 612 Sand	2.0	1.32
No. 48 Sand	3.15 mm.	1.27

Screen meshes tested included:

Mesh Screen Designation	Screen Opening microns/inches		Tyler Screen Scale Equivalent (mesh)	Open Area (%)
No. 3	6350	0.025	3	57.6
No. 20	841	0.0331	20	43.6
No. 40	420	0.0165	35	43.6

The filter tests were directed to determine the degree of treatment that could be achieved by using different depths and composition of filter media when operating at different flux rates, with and without the application of coagulants and polyelectrolytes. Using the results of the tests, criteria could be established to determine design parameters of full scale installations.

The principal water quality parameters carefully observed and recorded were: suspended solids, BOD, and COD. Measurements were also made on pH, temperature, total solids, settleable solids, coliforms, and total organic carbon. The laboratory analyses were performed by a local laboratory in Cleveland.

Filtration operational factors measured and recorded were: media depth and composition, flux rate, head loss, length of

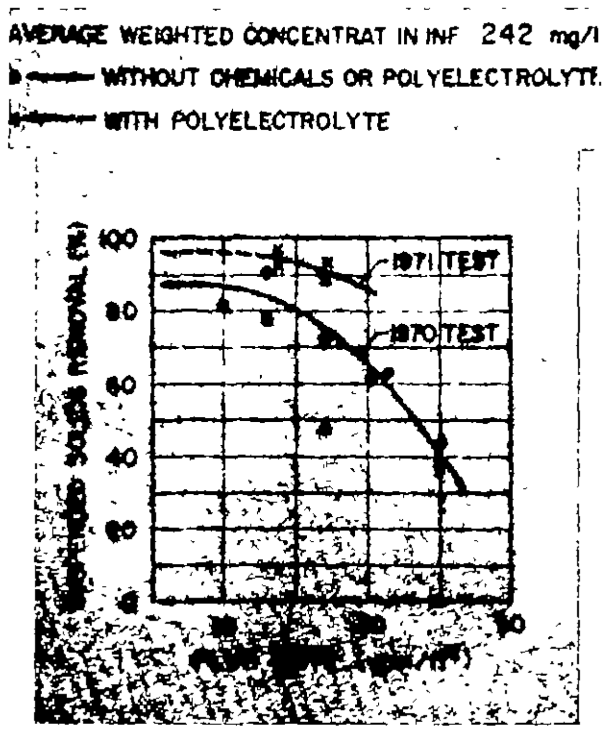

Figure 3
Filtration System
Performance--
Suspended Solids
Removal

Figure 4
Filtration System
Performance -
B.O.D. Removal